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SEISMIC VULNERABILITY OF MASONRY STRUCTURES THROUGH A MECHANICAL-BASED APPROACH

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Abstract. Enhancing the territorial resilience to natural events, such as earthquakes, is assuming a primary role in the current political debate. In the context of Disaster Risk Management, developing reliable vulnerability models for the seismic risk assessment at a territorial scale is an aspect of crucial importance. In this perspective, the paper presents a mechanical-based method for the evaluation of local-scale seismic fragility curves for unreinforced masonry buildings, based on the exposure data collected in the Italian CARTIS database. It uses a bidimensional finite element model and static nonlinear analyses to obtain the structural behaviour. Monte Carlo simulations are performed to propagate the uncertainties. Both local and global scale structural behaviour are considered to define the damage grade. A case-study regarding the city centre of Cosenza, in southern Italy, validates the proposal.

1 Introduction

As demonstrated by the latest events, like L'Aquila 2009, Central Italy 2016, Ischia 2017, Italy is a country with a high seismic risk, because of a high value of hazard which characterises the majority of the country and because of the high vulnerability of the building heritage, frequently not designed to respond to seismic actions due to a late seismic classification of the national territory. In recent years, the attention of Italian and European institutions to ensure greater safety of citizens against calamitous events, such as earthquakes, has increased.

In literature, the methods for the evaluation of seismic fragility curves are of three types, namely empirical, analytical or hybrid. The empirical approach defines statistical correlations among typological building characteristics, damage level and hazard values, based on information collected from past seismic events [2, 10]. The analytical approach defines the vulnerability through mechanical analyses that studies the damage evolution of a building, with assigned typological and structural characteristics, by increasing the hazard value [20, 1, 6]. The hybrid approach combines post-earthquake damage statistics with data simulated through numerical models of the building typology.

Starting from the modelling and analysis techniques, different mechanical seismic vulnerability methods for masonry structures have been presented. Lagomarsino and Cattari [11] proposed a mechanical model for the derivation of fragility curves based on a simplified model of the in-plane behaviour calibrated on an equivalent frame model. Uncertainties of different nature are considered as independent and propagated using the response surface method. A mechanical vulnerability assessment strategy using Monte Carlo simulations is proposed by Rota et al. [18]. A combination of pushover and nonlinear dynamic analyses is employed to define the fragility functions. Kappos et al. [9] proposed a hybrid method in which empirical data are used in conjunction with pushover analyses on equivalent frame buildings to construct fragility functions. Recently, an approach based on the vulnerability index and static nonlinear analyses and an equivalent frame of the masonry structure has been proposed [17]. Fragility functions which consider the out-of-plane behaviour of masonry structures using a failure mechanism approach are presented in [23]. Masonry structures, for instance, can be modelled by using simplified analytical solutions [23] or more advanced models that make use of the Finite Element Method (FEM). In this last case, great accuracy is achieved by micro-models which describe the single component of the walls (brick, mortar and their interface) and take into account the distribution of the mortar joints [5]. On the other hand, macro-models are also adopted to reduce the computational effort [19]. Such models consider the masonry as a homogenized material and can be applied to discretize the structure into three-dimensional, bi-dimensional or one-dimensional Finite Elements (FE). In this last group one finds equivalent frame approach that was formulated for the global response of masonry building to allow three-dimensional modelling by using beam elements [12].

This paper aims to show a hybrid approach aimed at developing local fragility curves of buildings, valid for specific building typologies, recognised and characterised through a first level exposure analysis conducted with the aid of the CARTIS form, developed in the framework of Work Package 2 "Inventory of existing structures and building typologies - CARTIS" of RELUIS Project promoted and funded by Italian DPC [21]. The peculiarity of the method, named HAREAS (Hybrid Analysis for the REgional-scale Assessment of Seismic vulnerability) consists in the possibility to develop a vulnerability model for masonry buildings at sub-municipal scale.

The mechanical model used gives a compromise between accuracy and efficiency. It is implemented in the code POR-2000 by Newsoft [16] and is based on a bidimensional discretisation using a hybrid-stress FE that captures the key features of the global behaviour of masonry structures at a reduced computational cost. The global structural response is obtained by performing static nonlinear analyses for varying directions and shapes of the seismic action. Simplified out-plane collapse mechanisms are also studied to account for the local-scale behaviour. To define the fragility curves, a virtual sample of buildings is generated using the Monte Carlo method. Each building is subject to global and local analysis, thereby obtaining the evolution of its static and kinematic configuration with PGA. These results are heuristically correlated with the damage scale reported in the European Macroseismic Scale 1998 (EMS-98) [7]. The attribution of the damage level considers both the global and the local behaviour. Finally, knowing the correspondence between the PGA values that generate the investigated level of damage, a set of fragility curves for each vulnerability class is built.

The paper is organised as follows; in Section 2, exposure analysis by CARTIS form is shown; in Section 3, the vulnerability classes assignment by SAVE procedure is illustrated; in Section

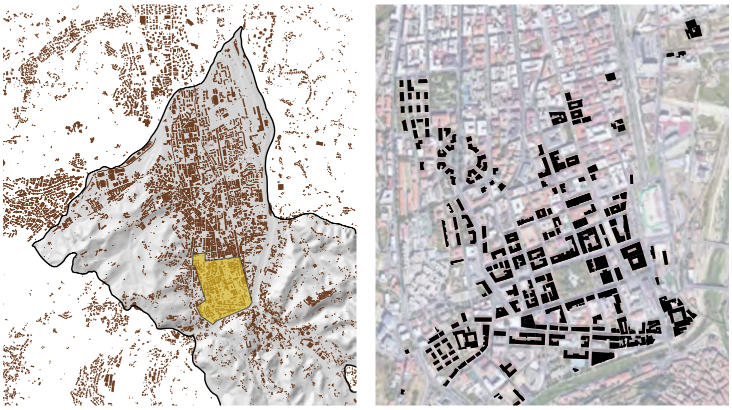

Figure 1: Location of the studied district on Cosenza urban area (left-hand side) and, in black, the masonry structures considered in the seismic vulnerability assessment (right-hand side).

Figure 2: Some pictures of the studied masonry structures.

4, some details on the numerical analyses of masonry structures are given; in Section 5, the adopted model of fragility curves is proposed; in Section 6, the case study of Cosenza city centre is presented. Finally, conclusions are drawn in Section 7.

2 Exposure analysis through CARTIS survey activity

2.1 Survey activity

In the methodology, the exposure assessment is developed through the data collected according to the CARTIS approach. CARTIS aims to furnish information collected by expert judgment at sub-municipal scale (and not at building scale), with the aim to collect data covering a large part of the regional territory in the perspective to develop regional characterizations of Italian ordinary buildings more reliable than simple census data [21].

2.2 Exposure model of Cosenza city centre

Mechanical based fragility curves are evaluated for unreinforced masonry buildings in the centre of Cosenza. The buildings under consideration have all been constructed after 1920 and nowadays are surrounded by newer constructions in reinforced concrete. Figure 1 shows a map of the analysed district and highlights the buildings under consideration. The total number of studied structures is 261.

The vertical structure is characterised by irregular masonry with horizontal brick lines about

Rectangular plan "C" shaped plan "O" shaped plan "L" shaped plan

Figure 3: Plan shapes and dimensions.

every 50 cm. All the buildings are insulated, but can present plan and elevation irregularity. The connection between orthogonal walls is good. Since the district under consideration is located on the city centre, the state of maintenance is decent to excellent and the destination is mainly for residential use.

The survey stage is divided into two parts. First, some features are obtained for all the buildings. Then, using Cartis form, more detailed information is obtained on a small number of buildings.

A GIS environment is useful to handle the information in the first-level survey which collects the following data: plan dimensions, number of storeys., underground levels, irregularity in elevation.

The second-level survey uses the Cartis form to collect the following data

1. Type of masonry;
2. Horizontal structure;
3. Roof features (thrusting, material);
4. Presence of tie-rods;
5. Inter-storey height;
6. Wall width at groud floor;
7. Percentage area of openings;
8. Mean wall length.

The additional data collected in the Cartis form is not used in this work. However, if some peculiar features arise from the analysis of the Cartis forms, then they can be modelled in the proposed mechanical-based method. Cartis form were collected for the 8% of the building under consideration.

2.2.1 Statistic elaboration of survey data

The data obtained in the first-level survey are known for every building, even if random uncertainties are present. Conversely, the data from the second-level survey are known only for a few buildings. In Tables 1 the results of the second-level survey are given.

Table 1: Frequency distributions of some building parameters.

property	mean	standard deviation	distribution
inter-storey height [m]	4.20	0.67	normal
external walls width [m]	0.75	0.10	uniform
internal walls width [m]	0.60	0.10	normal
walls length [m]	5.00	0.30	uniform
facade openings [%]	15	7	uniform

3 Vulnerability class assessment

Vulnerability assessment strategies are based on grouping the buildings in vulnerability classes in which buildings with similar behaviour are grouped using the SAVE method[22].

SAVE assigns the vulnerability class, in accordance with EMS-98 [7], considering its vertical structure and others typological and structural characteristics of the building influencing the response, such as presence of tie rods, number of stories, etc.

We refer to Zuccaro and Cacace [22] for the values of the weights and of the non-correlation coefficients and for further insights into SAVE.

4 Numerical analysis of masonry structures

The analysis of masonry structures is carried out at two scale levels. First, the global-scale behaviour is analysed under the hypothesis that masonry walls are well-connected each other. In doing so, the masonry structure is discretised into bidimensional FE. The structural response is obtained using nonlinear static analysis performed for multiple directions of the seismic action. Then, a local-scale analysis is conducted with the aim of identifying local collapse mechanisms. The result of global and local scale analyses is a relationship between seismic intensities and damage levels.

The adopted numerical framework is implemented in the software POR-2000 [16], developed by Newsoft. Details can be found in the user manual [16] and in the cited reference.

4.1 Global-scale analysis

4.1.1 Finite element modelling

The global-scale analysis is based on a FE macro-modelling of the building, in which the masonry is described as a homogenised material. Due to the hypothesis of good connection between walls, the slabs are supposed to be behave rigidly on their plane and only the in-plane resistance of the masonry walls is considered.

The adopted FE, named Flex-6m, is mixed, namely the displacement and the stress fields are interpolated independently. An important feature of Flex-6m is that the assumed stress field a-priori satisfies the equilibrium equations for zero bulk loads [14, 15, 13]. This choice ensures good accuracy, even with a rough discretisation in both the linear–elastic [15] and nonlinear range [13].

The nonlinear behaviour, developed in a context of non-associated plasticity, is based on assuming a set of planes on the FE where frictional response can take place. In addition, tensile and compression limit stress are considered. A simplified damage model is considered. It consists

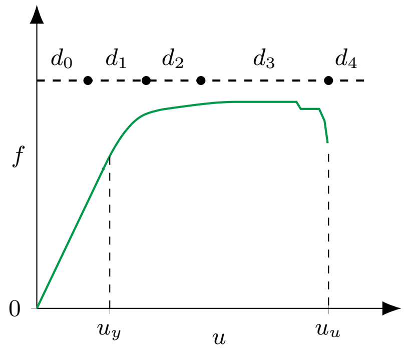

Figure 4: Capacity curve of a global-scale analysis and definition of the damage levels.

in zeroing the resistance of a masonry wall when the horizontal displacement reaches a ductility limit. Two different values of the ductility limit are considered, depending on which mechanisms is activated between in-plane shear or bending.

4.1.2 Nonlinear analysis

On the basis of a numerical model discretised by Flex-6m FE, the seismic response of the building is obtained through a nonlinear static analysis. First, a shape of the horizontal forces that model the seismic-induced loads is assumed. Then, the unitary load is scaled by an amplification factor f. The relation between the displacements of the structure, $\boldsymbol{u}$, and f defines the equilibrium path of the structure that is recovered pointwise using a path-following algorithm.

From the equilibrium path, it is possible to evaluate the capacity curve of the structure, that represents the evolution of the applied load with a reference displacement that is evaluated through an equivalence with the strain energy, see Fig. 4. Along this curve it is useful to identify two points, namely the elastic limit, having displacement u_y, and the ultimate point at displacement u_u. The nonlinear analysis is repeated using two different shapes of the horizontal forces, namely a constant and a linear distribution with the building height. Moreover, the analysis is repeated for 8 different directions of the seismic load. For each structure, the nonlinear analysis is therefore repeated 16 times.

4.2 Local-scale analysis

It is well-known that masonry structures can frequently undergo local collapse. Therefore, for each panel a simplified local-scale analysis is conducted accounting for the most frequent local collapse mechanisms. The analysis, whose details can be found elsewhere [16, 23], provides for each structure and for each direction of the seismic load, the value of a_g that causes the activation of the first mechanism, that is defined as a_g^l.

4.3 Definition of the damage grades

The evaluations of fragility functions requires the definition of a damage scale. The damage grades are chosen in accordance with EMS-98 [7], namely

- d_0: no damage
- d_1: slight
- d_2: moderate
- d_3: extensive
- d_4: near collapse
- d_5: collapse.

When mechanical-based methods are used, a numerical interpretation of EMS-98 damage grades is required. To this end, a heuristic approach is herein adopted. With respect to the global-scale behaviour, the damage grades from d_1 to d_4 are identified along the capacity curve (see Fig. 4) with the displacements u_{di}, $i = 1, \ldots, 4$, defined as[11]

$$\bar{u}_{d1} = 0.7u_y, \quad \bar{u}_{d1} = c_2 u_y, \quad \bar{u}_{d3} = c_3 u_u + (1 - c_3)u_y, \quad \bar{u}_{d4} = u_u, \tag{1}$$

where c_2 is and c_3 are two coefficients, with c_2 between 1.2 and 2 and c_3 between 0.3 and 0.5. The mechanical model is not capable of capturing the destruction of the building, namely d_5.

The seismic action is described by a response spectrum.In order to build fragility curves it is necessary to identify the seismic acceleration a_g that causes each damage grade, defined as $a^g_{g,i}$, for $i = 1, \ldots, 4$. To this end, we use the N2 method proposed by Fajfar [4] that allows to compare the response spectrum with the capacity curve.

Concerning the damage due to local collapse mechanisms, the EMS-98 grades are identified following the hybrid procedure presented in D'Ayala [3]. It has been shown [23] how the seismic acceleration that causes a local collapse obtained though a simplified local mechanism analysis corresponds to the d_3 damage grade. Different damage grades are then extrapolated from that value as

$$a^l_{g,i} = a^l_{g,i+1} \frac{i}{i+1} \alpha \qquad \text{if} \quad i \leq 2 \tag{2}$$

$$a^l_{g,i} = a^l_g \qquad \text{if} \quad i = 3 \tag{3}$$

$$a^l_{g,i} = a^l_{g,i-1} \frac{i}{i-1} \beta \qquad \text{if} \quad i > 3 \tag{4}$$

where the two coefficients assume the values $\alpha = 0.985$ e $\beta = 1.33$, but can be modified if empirical data is available.

Eventually, the value of a_g that causes each damage grade is the minimum between that related to global and local scale behaviour, namely

$$a_{g,i} = min(a^g_{g,i}, a^l_{g,i}), \qquad i = 1, \ldots, 4. \tag{5}$$

The damage d_5 is extrapolated from the other damage levels, as explained in the next Section.

5 Mechanical-based fragility curves

The fragility curves are functions of the intensity measure a_g giving the probability p that each damage state d_i, $i = 1, \ldots, 5$ is reached or exceeded. The fragility curves are well-fitted by a cumulative lognormal distribution [11, 24]

$$p(d \geq d_i) = \Phi \left(\frac{log\left(\frac{\lambda_i}{a_g}\right)}{\beta_i} \right), \quad i = 1, \ldots, 5, \tag{6}$$

where d is the structural damage, Φ is a normal cumulative probability function, while λ_i and β_i are the mean value and the logarithmic standard deviation of the a_g values that cause the d_i damage grade. For each damage grade d_i, the definition of a fragility curve requires the evaluation of the two parameters λ_i and β_i. To this end, it is necessary to obtain a population of buildings reflecting the in-situ variation of the significant features of the masonry buildings under consideration and to model the uncertainties involved in the seismic vulnerability.

The discrete probability histograms, which give, for a fixed value of a_g, the probability that the structural damage is equal to each i-th damage level, are obtained from the fragility functions as

$$\begin{aligned} p(d = d_5) =& p(d \geq d_5) \\ p(d = d_i) =& p(d \geq d_i) - p(d \geq d_{i+1}) \quad i = 4, \ldots, 1 \\ p(d = d_0) =& 1 - p(d \geq d_1). \end{aligned} \tag{7}$$

Because the numerical method is not capable of giving the value of a_g that produces the damage grade d_5, the relative fragility curve is extrapolated from that evaluated for the damage grade d_4. Therefore, following Lagomarsino e Cattari [11], the probability that the damage exceeds d_5 is

$$p(d \geq d_5) = 0.8 \left(1 - \left(1 - 0.14 s_d^{1.4}\right)^{0.35}\right) p_{d4}, \qquad s_d = \sum_{i=1}^{4} i \cdot p(d \geq d_i). \tag{8}$$

Three uncertainty groups are considered, namely: i) structural response; ii) damage level definition; iii) response spectrum shape [11].

6 Fragility curves for masonry buildings in Cosenza

In this section the proposed method is applied to a case study regarding the masonry buildings in Cosenza city centre. First, the random generation of the building population using the Monte Carlo method is presented. Then, the response spectrum affected by uncertainties is evaluated through the analysis of recorded seismic inputs. Finally, the fragility curves are presented and compared with empirical state-of-art curves.

6.1 Monte Carlo generation of a building population

The two-level based survey provides deterministic information for some data and frequency distributions of some others. On the basis of such information, it is necessary to construct a building population to be analysed with POR-2000. The starting point is the database of buildings given by the first-level survey. Then, the frequency distribution of the structural parameters is assumed to be valid for all the buildings.

6.1.1 Geometry

Starting from the plan dimensions, we evaluate a probable disposition for the internal walls. Each edge, of length L_d, is modified by summing a random number that takes into account the random uncertainty in the measures. Then, each edge is randomly divided into parts by imposing that the wall distance is in accordance with limits given in Table 1.

Some randomly generated models are shown in Figure 5.

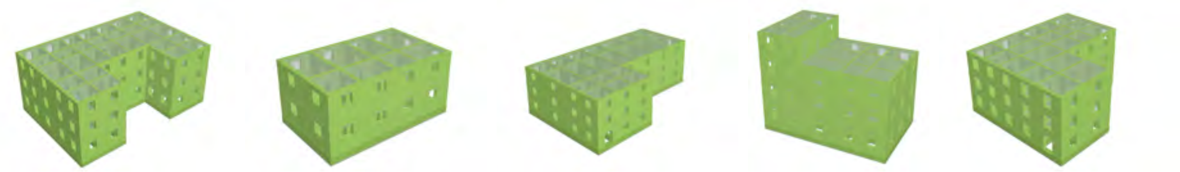

Figure 5: Some randomly generated building models.

6.1.2 Mechanical properties

The masonry type is only known qualitatively. The Italian code [8] provides ranges of the homogenised mechanical properties of the masonry for a given description of its main features. This information is useful when no in-situ test results are available.

6.2 Fragility curves

In this section we present the fragility curves constructed for the masonry buildings in Cosenza city centre. The Monte Carlo simulation is conducted for a population of $n_b = 10000$ buildings. For each buildings, the nonlinear analysis is repeated 16 times, changing the action shape and direction. Then, the N2 method is performed, for each capacity curve, $n_2 = 200$ times in order to take into account the uncertainties on the damage grade definitions and on the response spectrum shape.

After applying SAVE, the major part of the building are in classes B or C (79%) and only a small fraction in class A (21%). Since the behaviour of class B an C buildings is quite similar, a unique class is considered which is named B-C. Class A buildings are characterised by the absence of tie-rods, high number of levels and irregularity. In such buildings local collapse mechanisms are likely to happen for low values of seismic intensity. Conversely, class B-C buildings have tie-rods or rigid slabs and a global collapse mechanism is expected.

Figure 6 shows the fragility curves obtained for the two vulnerability classes.Class A fragility curves are characterised by higher dispersions and lower mean values that class B-C. This is due to the local-scale collapse mechanisms that characterise the structural behaviour of class A buildings and produce severe damages for low values of the seismic acceleration. Moreover, the contemporary presence of local and global scale damages increases the dispersion of fragility curves, since two different analysis methods are employed. On the other hand, class B-C buildings reach the damage levels almost exclusively for global-scale behaviour. These buildings provide a better response against the seismic action and are less sensitive to the variation of the parameters.

The proposed mechanical-based fragility curves are compared with empirical curves in Fig. 6. In particular, comparison is made with the fragility curves recently proposed by Perelli et al. [24] who employed SAVE method as vulnerability classifier. A close correspondence is verified for class A buildings. More distinct differences are observed on class B-C fragility curves, even if a similar trend can be recognised for all the damage levels.

7 Conclusions

A mechanical-based vulnerability assessment method for unreinforced masonry structures has been proposed. It is based on Monte Carlo simulations to evaluate the propagation of

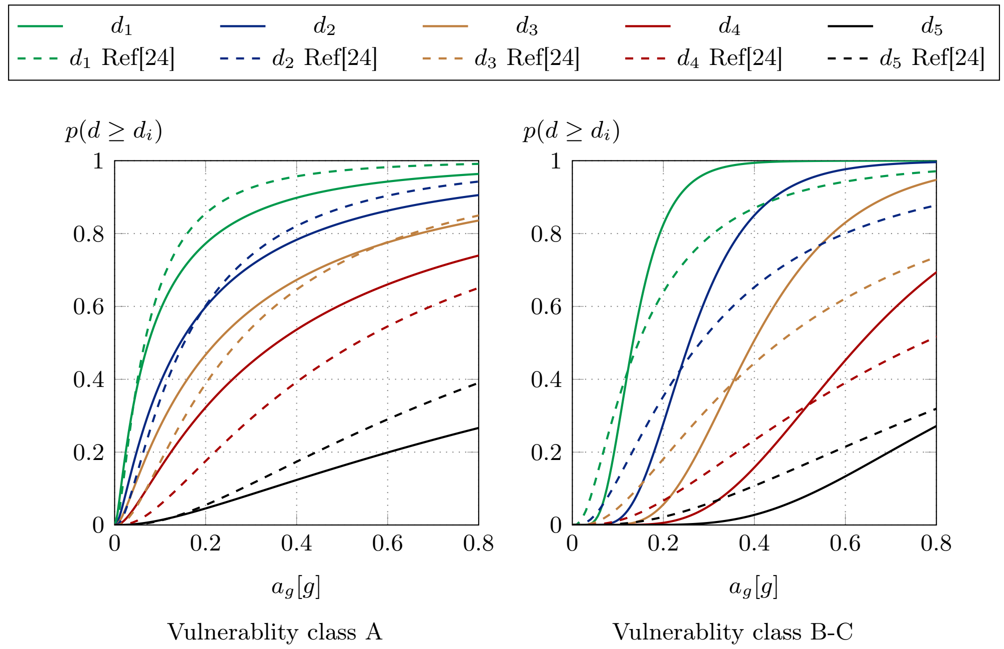

Figure 6: Fragility curves for the masonry buildings in Cosenza, subdivided for vulnerability classes, compared with the empirical functions proposed by Perelli et al. [24].

the uncertainties involved in the definition of the seismic risk. Global and local scale collapse mechanisms are considered in the evaluation of the fragility curves. The structural behaviour is obtained using a commercial software, namely POR-2000. It is based on an accurate model of masonry walls using hybrid-stress bidimensional finite elements. Nonlinear static analyses, repeated for multiple directions of the seismic action, are performed to evaluate the building capacity. The analysis is highly efficient, thereby allowing thousands of runs to be executed in little time.

Cartis database is employed to extract data on the buildings, together with a GIS environment. The damage grades are in accordance with EMS-98 definitions and the peak ground acceleration is adopted as intensity measure. The proposed method is applied to unreinforced masonry buildings located in the city centre of Cosenza. Fragility curves are obtained for two vulnerability classes and are compared with state-of-art empirical results.

The proposed method demonstrates the possibility of performing vulnerability analyses at regional scale and stands as a valid tool that balances accuracy and computational cost.

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